U.S. Army Corps of Engineers
Seattle District

Centralia Flood Damage Reduction Project
Chehalis River, Washington
General Reevaluation Report

Prepared for:

Lewis County, Washington
Corps of Engineers, Seattle District

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Appendix B:
Skookumchuck Dam Modifications

July 2002

Table of Contents

1. Introduction

1.1 General

This technical memorandum summarizes the reconnaissance level analysis of modifications to the existing Skookumchuck Dam for the purpose of flood control. Modifications to Skookumchuck Dam are being considered as part of the Centralia, Washington Flood Damage Reduction Project. The other potential elements and alternatives of the project are discussed in separate technical reports. The alternatives being studied for the project are listed below. Each alternative has its own corresponding technical report. A discussion of the hydraulic modeling used to evaluate the various project components and alternatives, including modifications to Skookumchuck Dam, is presented in Technical Report No. 8. A summary of the modeling results may also be found in the Hydrology and Hydraulis Appendix to the USACE General Reevaluation Report.

The Skookumchuck Dam modifications could provide flood control storage of 11,000 to 20,000 acre-feet, and could significantly reduce flood stages along the Skookumchuck River floodplain. For the U.S. Army Corps of Engineers (USACE) developed synthetic 100-year hydrograph, peak flood stages would be reduced in the town of Bucoda by approximately 1.78 feet for the 11,000 acre-feet alternative, and 3.22 feet for the 20,000 acre-feet alternative when the dam modifications are combined with the USACE recommended levee arrangement. In the town of Centralia, the peak flood stages would be reduced approximately 0.79 feet for the 11,000 acre-feet alternative, and 1.37’ for the 20,000 acre-feet alternative.

The pre-feasibility analysis indicated that modifications to Skookumchuck Dam would provide the most cost effective flood control storage. In addition, modifications to Skookumchuck Dam would have the least environmental impact of all the storage dam alternatives previously considered. While modifications to Skookumchuck Dam do not result in significant flood stage reductions on the main stem of the Chehalis River, the dam is an essential component to the overall Project. The flood control storage provided by the dam aids in reducing flood stages along the Skookumchuck River, as well as offsets any increases caused by the flood reduction measures in the Centralia-Chehalis area.

2. Existing Conditions

2.1 General

Skookumchuck Dam is located on the Skookumchuck River about 12 miles northeast of Centralia, Washington, at Skookumchuck river mile (RM) 21.9. The dam was constructed in 1970 to supply cooling water to the coal-fired Centralia steam electric plant. The dam has a rolled earthfill central core, buttressed by an earth and rockfill shell. The structure is approximately 190 feet high, with the top of the dam at elevation 497 feet. All elevations referred to in this report are based on NGVD 29 with the 1947 adjustment. See Plate 1 for a location map.

The dam has a 130-foot wide uncontrolled side-channel spillway in a rock cut on the left abutment. The spillway is a concrete ogee with a crest at elevation 477 feet. The spillway invert is at elevation 465 feet. Water passes over the ogee and spills into a 130-foot long by 40-foot wide concrete lined trough. Water then spills down a concrete lined chute. The chute is almost 600 feet long and has a bottom slope that varies from 17% to 25%. The spillway chute ends in a stilling basin that directs the discharge into a rock cut leading back to the natural river channel. Facilities are located adjacent to the stilling basin to trap migrating salmon and steelhead for truck transport over the dam. See Plates 2, 3 and 4 for a plan and sections of the dam and spillway.

During low flow months, water released from storage travels downstream to a diversion pumping station at RM 7.3. From there water is pumped through a 3-mile pipeline to the steam electric plant. Under an agreement between the dam owner and state agencies, additional releases are made from the reservoir to supplement flows in the Skookumchuck River to improve fishery habitat.

Outflow from the reservoir is either over the spillway crest at elevation 477 feet, or through the outlet works. The existing outlet works consist of an inclined, multilevel intake structure that connects to the construction diversion tunnel and discharges through two 24-inch Howell-Bunger valves into the spillway stilling basin. The intake gates are set at elevations 449, 420, and 378 feet. The discharge capacity of the outlet works is approximately 220 cubic-feet-per-second (cfs) when the pool elevation is at the spillway crest.

Storage behind the dam is essentially a fill and spill operation. The limited outlet capacity of the dam causes the reservoir to fill to the spillway crest at elevation 477 feet early in the flood runoff season.

Once the reservoir is full, flood inflow to the reservoir passes over the un-gated spillway, which was originally designed for a discharge capacity of 28,000 cfs with the reservoir pool at elevation 492 feet.

Storage capacity of the reservoir between the normal minimum pool at elevation 400 feet and the spillway crest at elevation 477 feet is 38,700 acre-feet. Additional usable storage of 3,170 acre-feet is available between elevations 378 feet and 400 feet. Dead storage is approximately 1,420 acre-feet between elevations 378 and the base of the dam.

Preliminary investigations by the U.S. Army Corps of Engineers (USACE) indicated that flood control storage at Skookumchuck Dam could be feasible without jeopardizing the steam plant water supply. The USACE investigated several alternatives for modifications, which are presented in detail in the USACE's December 1982 and February 1992 reports (USACE, 1982, 1992).

2.2 Probable Maximum Flood

The probable Maximum Flood (PMF) is defined as the flood that could be expected to occur from the most severe combination of hydrometeorological conditions reasonably possible in the region. The existing spillway was originally designed to pass a peak PMF outflow of 28,000cfs at a maximum reservoir pool elevation of 492 feet, with a freeboard of five feet to the top of the dam at elevation 497 feet. A PMF analysis was performed for this study to verify or revise the PMF value of 28,000 cfs used in the original design of the existing spillway. Bechtel Civil & Mineral performed the original PMF analysis and spillway design in the late 1960's.

The revised PMF was derived by using the HEC-1 computer program applying a Clark's hourly unit hydrograph to the PMP plus snowmelt excess while accounting for baseflow. The PMP was determined from HMR 57 (**Hydrometeorological Report 57**, *Probable Maximum Precipitation - Pacific Northwest States; National Weather Service; October 1994*) to be 24.73 inches for a 72-hour November-February general storm. December snowmelt was used as the December persisting dew points and realistic snowpack would produce the highest snowmelt during the PMP. Snowmelt during the PMP storm was computed to be 7.44 inches using procedures outlined in EM 1110-2-1406, "Runoff from Snowmelt". It was assumed that there would be a 75 percent availability of the computed snowmelt, or 5.58 inches of snowmelt during the December PMP. Precipitation was distributed based on pattern 'e' in HMR 57.

The Clark Unit Hydrograph parameters and basin losses were calibrated from an optimization study based on observed events of January 1972, January and November 1990, and February 1996 at the streamgage near Vail (upstream of Skookumchuck Dam). Unit hydrograph parameters derived for the Vail gage were transposed to the dam by adjusting for travel time and the ratio of Tc to the attenuation constant R. Loss rates were considered the same at the gage and the dam site. The exponential loss rate parameters, and the Clark Unit Hydrograph parameters derived from the optimization studies were adjusted to reflect conditions associated with the larger PMF. Basin losses during the PMF were defined by a zero initial loss rate, assuming a saturated ground due to an antecedent storm, and a uniform loss rate of 0.05 inches per hour. The Clark Unit Hydrograph used in computing the PMF was specified by the Tc and R values of 5.02 and 7.52, respectively.

Base flow was estimated as the recession flow from a 100-year flood assumed to occur 3 days prior to the PMP storm. The estimated base flow had an initial value of 500 cfs and receded to approximately 30 cfs at 96 hours during the PMF. The spillway design flood (SDF) for the Skookumchuck Dam was determined by routing the PMF inflow through the reservoir and spillway. An initial reservoir elevation of 478 feet was used in routing the PMF through the reservoir based on antecedent flow conditions.

The results of analysis indicate that the PMF has a peak inflow of 32,500 cfs, a peak outflow from the current spillway of 30,600 cfs, and a mean 3-day inflow 15,000 cfs, or 89,500 acre-feet. The study also showed a maximum reservoir elevation of 492.68 feet, leaving a freeboard of 4.32 feet. These results indicate a higher PMF in comparison with the original spillway design PMF, which had a peak flow of 28,000 cfs. The original design PMF discharge, together with the calculated PMF discharge and reservoir elevations is shown in Figure 1. A study performed by Bechtel Civil & Mineral, Inc. for PacifiCorp in 1987 estimated a maximum reservoir wave run-up of 3.8 feet, 0.52-feet lower than the available freeboard of 4.32 feet during the PMF.

The USACE performed a preliminary review of the PMP and PMF calculations in January 2000, and an acceptance of the calculations was recommended. The new PMF was not routed through the various alternatives being considered for dam modification. The peak PMF outflow and peak reservoir pool level could change slightly depending on the spillway and outlet works modifications proposed. Additional analyses will need to be performed in the next phase of studies.

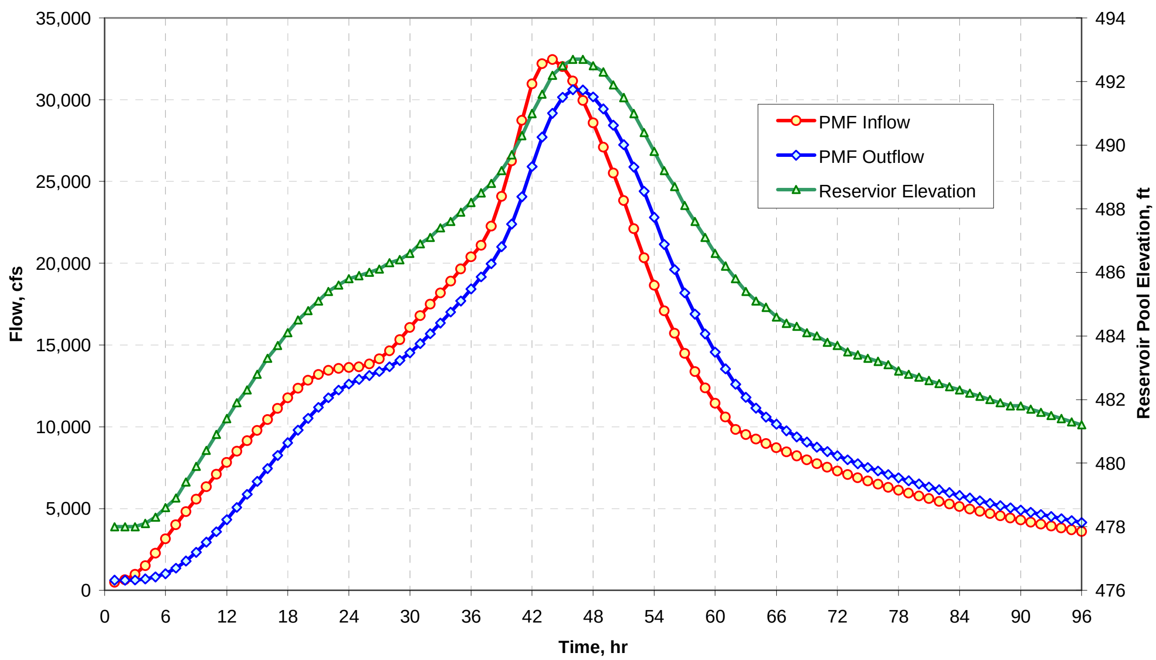


Figure 1: Peak PMF Outflow and Reservoir Elevation

2.3 Dam Safety Considerations

Any proposed modifications to Skookumchuck Dam must enable the project to safely pass the Probable Maximum Flood (PMF) at the maximum design pool. The dam embankment elevation must be sufficient to prevent overtopping during the PMF, while accounting for contingencies such as surcharge, wind wave runup, and embankment settlement. The dam embankment currently has a top elevation of 497 feet. The current maximum design pool level is at elevation 492 feet, and the current estimated peak reservoir pool level during PMF is at elevation 492.68 feet. The 4.32 feet between the estimated maximum pool elevation and the top of the dam was considered to be adequate freeboard for this study. More detailed analyses should be performed in the next phase of the study to determine the appropriate freeboard for the structure.

PacifiCorp (formerly Pacific Power & Light, the dam operator) had a dam safety and seismic stability analysis performed on the dam in 1988, which the USACE later reviewed. The USACE determined that, with the new operation for flood control, the embankment would suffer distress during the design earthquake, but would not fail and did not require any modification (USACE, 1992). More recently, PacifiCorp had a FERC (Federal Energy Regulatory Commission) Part 12 dam safety inspection performed in 1996. Stability analyses were performed for normal operating conditions, PMF, rapid drawdown, and seismic loading conditions. The embankment dam, spillway and all other structures were found to be safe for all cases investigated (Black & Veatch, 1996).

Due to uncertainties about the nature of the foundation materials and properties, USACE, PacifiCorp, and the FERC are currently reviewing foundation liquefaction and stability. The proposed changes to the reservoir operation for flood control will be taken into account as part of the study.

Other issues related to dam safety and operation could be any potential problems with debris or sediment. In discussions with the dam operating personnel, it was determined that there have not been any significant problems related to either sediment or debris in the operation of the spillway or outlet works. Additional investigations may need to be performed in the next phase of studies.

2.4 Reservoir Regulation Considerations

USACE developed a preliminary flood control operation rule curve as part of its flood control operations investigation (USACE, 1992). The USACE rule curve provided flood control storage of approximately 11,900 acre-feet between elevations 453 and 477 feet, from November 1 to February 1. After February 1, the reservoir would be allowed to refill. Drawdown of the reservoir would begin each year in early to mid September and would continue until elevation 453 feet was reached, usually around the first of November.

USACE performed a water supply study of the Skookumchuck reservoir as part of its investigation to determine if sufficient storage would be available to meet water supply and minimum instream flow requirements for fisheries and power diversion with storage operations for flood control (USACE, 1992). USACE assumed that PacifiCorp would divert its entire 81 cfs water right, and determined that minimum instream flow and water supply requirements could be met in all years with the USACE proposed flood control operation rule curve. The steam plant currently uses only up to 54 cfs for the two existing steam turbine units.

The flood control operation rule curve must also ensure releases in accordance with the existing fishery flow agreement. The agreement between PacifiCorp and Washington Department of Fish and Wildlife (WDFW) provides a minimum instream flow of 140 cfs from September 10 to October 31 for salmon spawning. Incubation flows begin on November 1, or at the completion of spawning, as determined by WDFW. A minimum of 95 cfs is supplied until March 31. From April 1 through August 31 rearing flows are set at a maximum of 95 cfs, or natural river flow plus 50 cfs, whichever is less. Rearing flows may be adjusted downward as determined by WDFW to preserve water for use during the spawning period. The instream flow agreement also provides for instream water temperatures of 50° to 55° F. These temperatures must be maintained, to the maximum extent possible, depending on reservoir and climatic conditions.

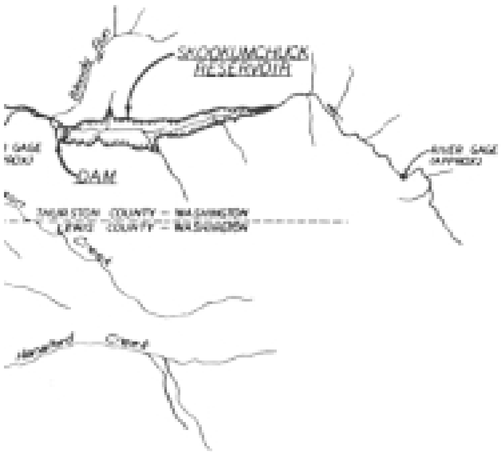
Flood regulation at the Skookumchuck Reservoir would seek to maintain the 4,900 cfs control flow at the Pearl St. Bridge in Centralia. Discharge would be reduced in increments of not more than 500 cfs per hour to a minimum flow of 95 cfs. Inflow would be stored until the routed releases plus local inflows do not exceed control flows and the possibility of adding to the Chehalis River peak has passed.

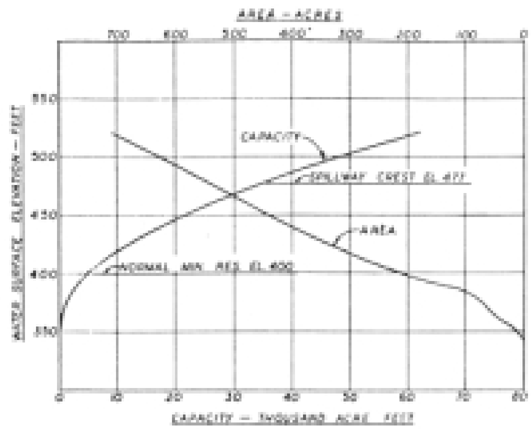
Reservoir evacuation should take place as soon as possible to provide storage for subsequent storm events. The reservoir would be evacuated by releasing outflows that, combined with local inflows, yield near control-flow levels. The evacuation releases would be greatly reduced as the minimum flood control level is approached.

Reservoir evacuation after a large storm would take 3 to 5 days. Consequently, a maximum outflow of 4,000 cfs may be achieved while maintaining river flows below control levels. Although a maximum discharge of 4,000 cfs may be desirable to minimize evacuation time, a discharge as low as 3,000 cfs would still meet evacuation requirements. A 3,000 cfs outlet capacity conforms to the guidelines in ER 1110-2-50 for establishing minimum reservoir outlet capacity for drawdown of lakes impounded by civil works projects. A low pool discharge capacity of at least 3,000 cfs would be required to evacuate Skookumchuck Reservoir from elevation 492 within five days.

A 3,000 csf outlet capacity at minimum reservoir pool was used in the earlier USACE study, and was assumed for the purposes of this study. The minimum reservoir pool was assumed to be at elevation 455 feet. It has also been assumed that low flow releases would continue to be made through the existing outlet works consisting of the multi-level intake and two 24-inch Howell-Bunger valves. The hydraulic design of the flood control outlet works, and the flood control regulation rule curves will need to be refined and finalized in the next phase of studies. In addition to hydraulic and engineering considerations, any downstream environmental effects related to reservoir operation and flood control regulation will need to be considered.

The dam modifications currently being proposed could provide, approximately, an additional 9,000 acre-feet of storage between pool El. 477 and El. 492. This additional storage could potentially be available to augment summer low flows downstream if it were determined that this would be environmentally beneficial. This would, however, require a change in the current reservoir conservation pool level and is not being proposed at this time for the flood reduction project. If this action were to be pursued in the future, any potential environmental impacts and dam safety issues associated with a higher conservation pool would need to be addressed.

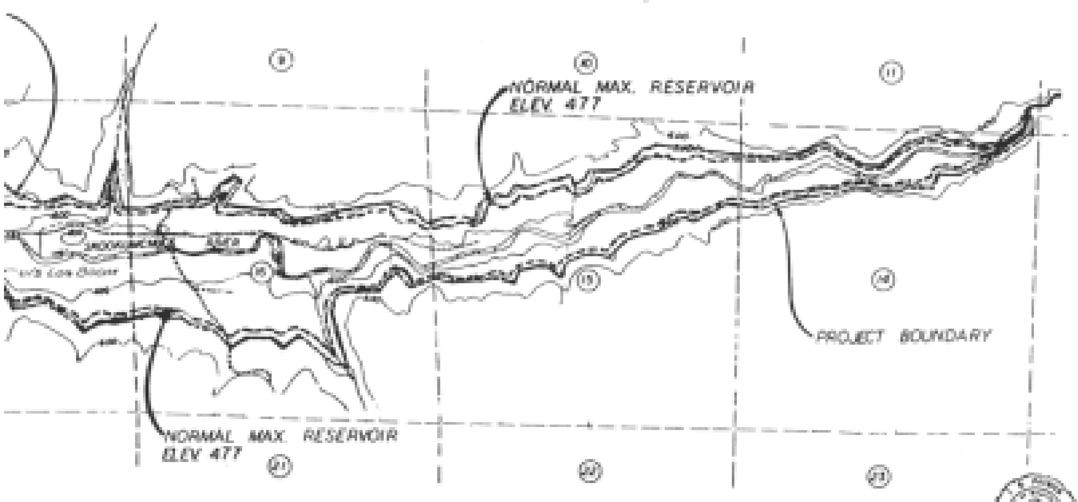




RESERVOIR AREA-CAPACITY CURVES

NOTES

1. Reservoir and Site Topography based on Topographic Maps prepared by H.G. Chickering Jr. by aerial photogrammetry in 1967.
2. Horizontal Datum, Washington Coordinate System (South Zone)
3. Vertical Datum, U.S. Coast & Geodetic Survey, North American 1947 Adjustment



UCK RESERVOIR AND PROJECT BOUNDARY
SCALE 1"=1000'

1 IN. = 1 MILE

1 IN. = 1000 FT.

AS BUILT

ISSUED FOR CONSTRUCTION

ISSUED FOR PREPARATION OF PROPOSALS

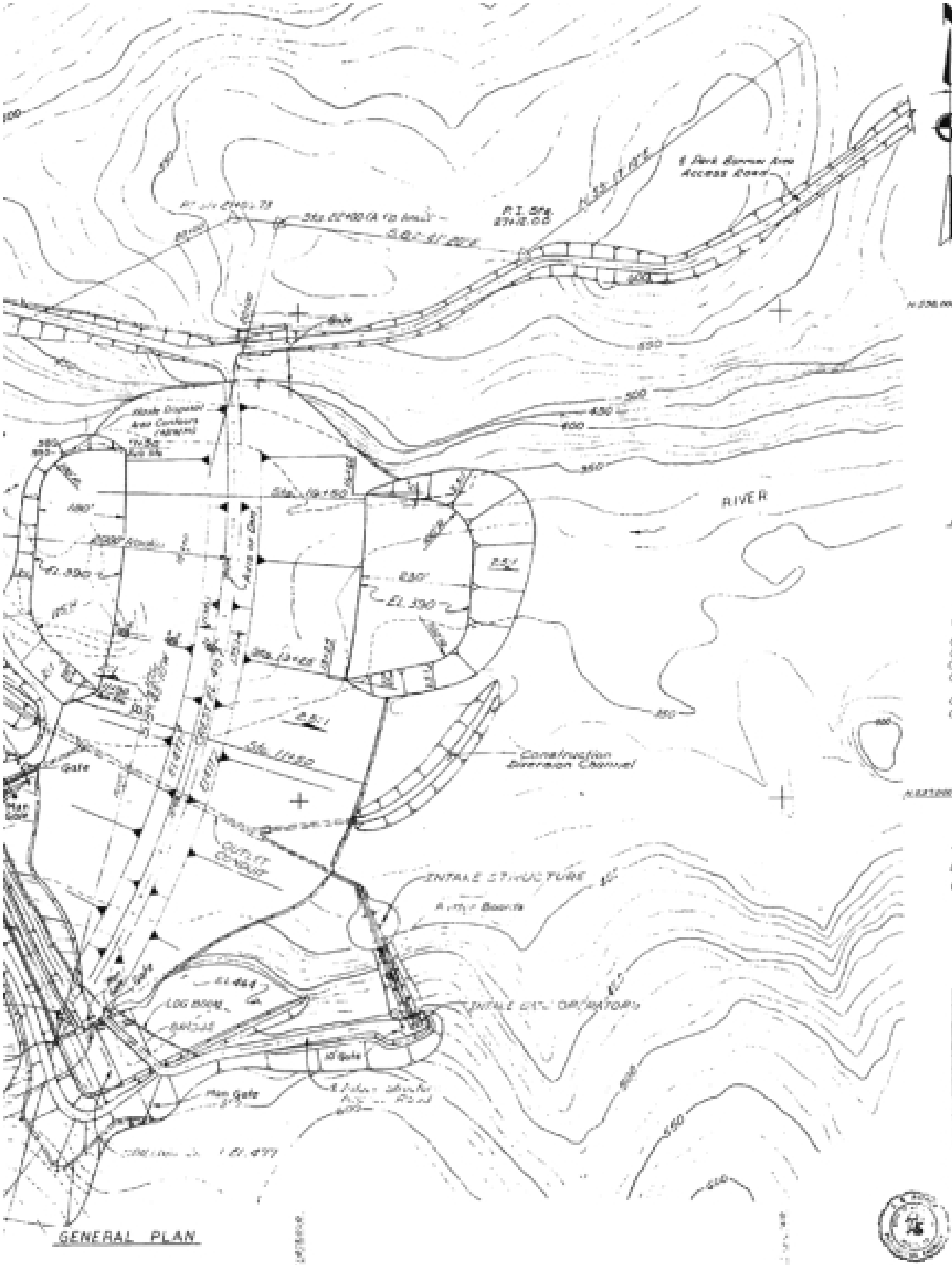
BECHTEL CORPORATION
SAN FRANCISCO

PACIFIC POWER & LIGHT COMPANY
and
THE WASHINGTON WATER POWER COMPANY
CENTRALIA POWER PLANT PROJECT

SKOOKUMCHUCK DAM AND RESERVOIR

PROJECT LOCATION AND RESERVOIR MAP

6442 C-5100



Revision Note 4

Revised Construction Diversion Channel and Dam Access Road. Added Park Borrow Area Road. Revised spillway cut slopes.

REFERENCE DRAWINGS

C-5100 Project Location & Reservoir Map
C-5130 Dam-Embankment Section & Profile
C-5131 Dam-Embankment Sections & Details
C-5140 Outlet Works-Arrangement, Plan Profiles & Typical Sections
C-5170 Spillway-General Arrangement
C-5194 Access Roads-Plan & Profile Sh. 2 of 2

LEGEND

△ — Survey Monument
⊗ — Embankment Piezometer

100 0 100 200 300 400 FT
1 IN. = 100 FT.

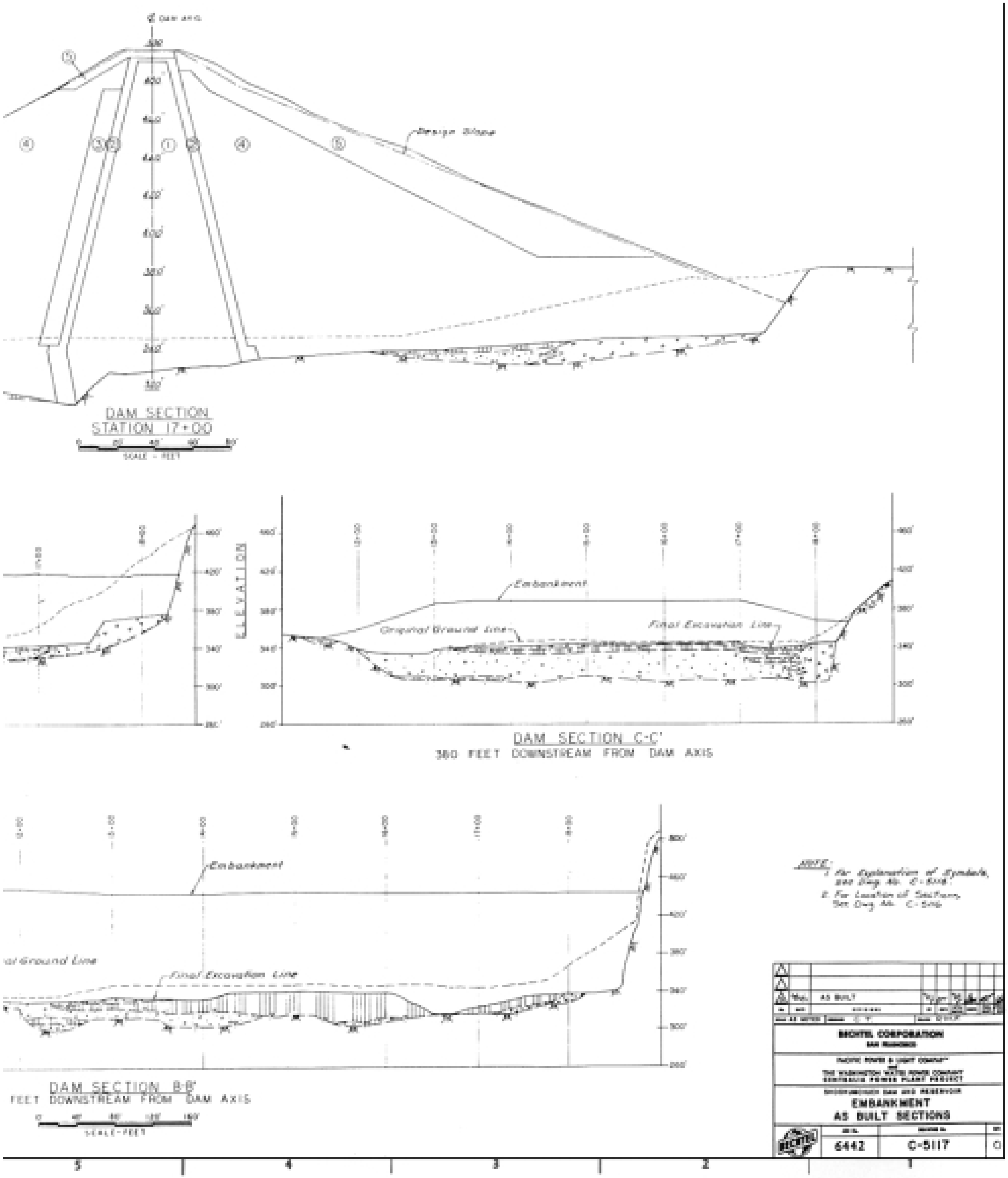
BECHTEL CORPORATION
SAN FRANCISCO

PACIFIC POWER & LIGHT COMPANY
AND
THE WASHINGTON WATER POWER COMPANY
CENTRALIA POWER PLANT PROJECT

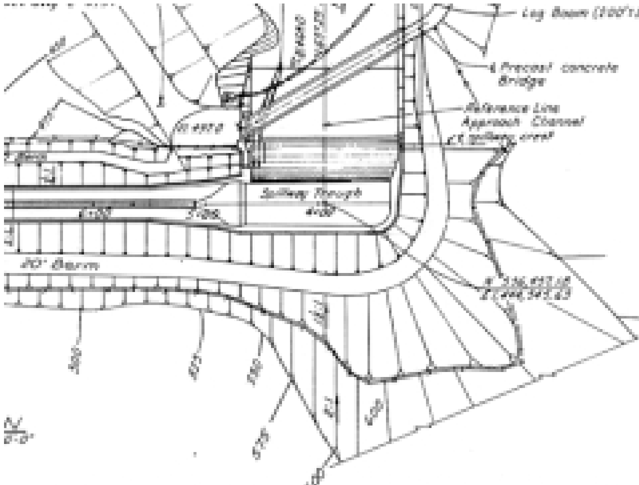
SKOOKUMCHUCK DAM AND RESERVOIR

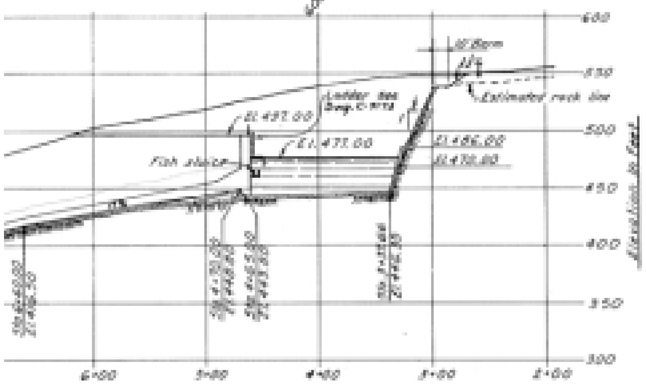
GENERAL PLAN

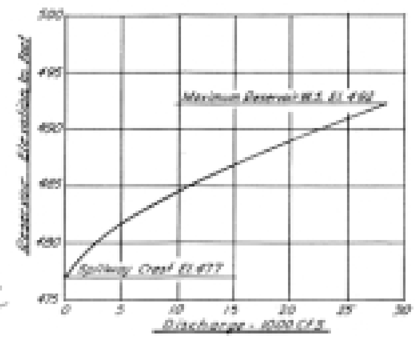
JOB No.	DRAWING No.	REV.
6442	C-5125	4



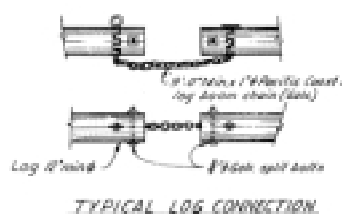
Design Slope
DAM SECTION
STATION 17+00
SCALE - FEET
ELEVATION
Embankment
Original Ground Line
Final Excavation Line
DAM SECTION C-C'
380 FEET DOWNSTREAM FROM DAM AXIS
Embankment
Final Excavation Line
DAM SECTION B-B'
FEET DOWNSTREAM FROM DAM AXIS
SCALE-FEET
NOTE:
1. For Explanation of Symbols, see Dwg. No. C-5116.
2. For Location of Sections, See Dwg. No. C-5116.
AS BUILT
BECHTEL CORPORATION
EMBANKMENT
AS BUILT SECTIONS
6442
C-5117







SPILLWAY RATING CURVE



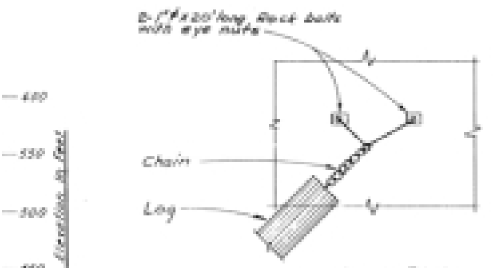
TYPICAL LOG CONNECTION
N.T.S.

NOTE

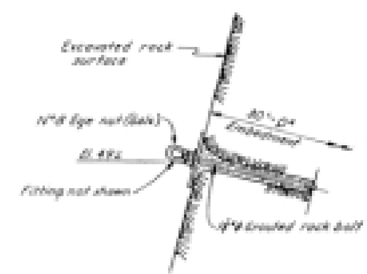
1. For location of log boom see dwg. C-5125

REFERENCE DRAWINGS

C-5125 General Plan
C-5151 Embankment - Sectional Details
C-5168 Outlet Structure & Fish Trap - General Arrangement
C-5171 Spillway - Conc. Outline - Approach, Crest & Trough, Sh. 1 of 2
C-5172 Spillway - Conc. Outline - Approach, Crest & Trough, Sh. 2 of 2
C-5173 Spillway - Conc. Outline - Retaining Walls
C-5174 Spillway - Conc. Outline - Fish Sluice
C-5175 Spillway - Conc. Outline - Chute
C-5176 Spillway - Conc. Outline - Stilling Basin
C-5180 Spillway - Excavation, Sh. 1 of 2
C-5181 Spillway - Excavation, Sh. 2 of 2
C-5182 Spillway - Miscellaneous Metal
C-5183 Bridge - Conc. Outline & Reinforcement, Sh. 1 of 2
C-5164 Outlet Structure and Fish Trap - Piping
E-5138 Electrical Layout - Spillway, Intake Structure



NORTH BANK



SOUTH BANK

APPROACH CHANNEL ANCHORAGE DETAILS
SPILLWAY LOG BOOM

1 IN. = 80 FT.

Revision Note 4:
Revised Approach Channel Anchorage Details, Spillway Log Boom, Chain Link Fence, Berm width & Cut slopes.

4		AS BUILT - REV.
		ISSUED FOR CONSTRUCTION
		ADDENDUM NO. 1
		ISSUED FOR PREPARATION OF PROPOSALS
		Issued for Approval of State of Wash. Dept. of Water Resources

BECHTEL CORPORATION
SAN FRANCISCO

PACIFIC POWER & LIGHT COMPANY
and
THE WASHINGTON WATER POWER COMPANY
CENTRALIA POWER PLANT PROJECT

SKOOKUMCHUCK DAM AND RESERVOIR

SPILLWAY
GENERAL ARRANGEMENT

JOB No.	DRAWING No.	REV.
6442	C-5170	4

3. Previous USACE Alternatives

3.1 General

Studies in the early 1980s by USACE proposed modifications to Skookumchuck Dam to provide flood control and regulation. The initial proposal suggested a separate intake tower with an open-channel tunnel in the right abutment along with addition of a 12-foot high steel bascule gate on the existing uncontrolled spillway. Further studies recommended deletion of the spillway gate due to reliability concerns of gate operation during floods. The additional studies also suggested modifications of the existing spillway to permit reservoir drawdown and withdrawal capability were more cost effective than construction of the right abutment tunnel.

3.2 Spillway Modifications

The USACE alternative incorporated two 10-foot high by 12-foot wide hydraulically operated slide gates into the existing spillway. An approximately 25-foot width of the existing spillway would be removed down to elevation 440 feet and reconstructed to incorporate the two slide gates. The gates were sized to discharge 3,000 cfs at a pool elevation of 452 feet with critical depth control (8 feet) at the gate entrance. The gates would be regulated to discharge no more than 3,000 cfs for pool elevations between 453 feet and 477 feet. The gates would be completely closed at pool elevations of 480 feet and greater, at which time all flow would pass over the existing uncontrolled spillway. Gate operating equipment would be enclosed in a cavity within the ogee crest. The exact location of the sluice along the existing spillway was not finalized.

The existing spillway chute and trough would be lowered approximately 10 feet in order to prevent a flow control shift from the gated entrances to the chute. The chute entrance would be lowered to elevation 438 feet and would have a width of 35 feet. With this geometry, the control would remain at the gates. The tailwater depth at the slide gates, for a discharge of 3,000 cfs, would be approximately 7 feet, which is one foot below the critical depth control at the gate.

From the chute entrance, the chute would slope at about 12 percent to meet the existing chute invert in a distance of about 200 feet. The chute walls are concrete lined 7 to 13 feet vertically above the invert, with excavated rock side slopes above the concrete lining. The chute lining was originally designed to be sufficient to contain a discharge of about 10,000cfs, which corresponds to approximately the 100-year flood peak discharge. The USACE proposed extending the concrete lining 4 to 13 vertical feet to fully contain the PMF. The spillway chute appears to have more than adequate freeboard to contain the PMF.

The construction cost of this alternative was estimated to be approximately $5,748,000 in 1989. The total project cost, including engineering and construction management, was estimated to be $11,928,000. Escalated to current 2002 price levels, using the ENR construction cost index, results in a total project cost of approximately $16,818,480.

USACE initially considered locating the outlets works on the right abutment. The scheme involved a freestanding tower intake and open channel tunnel through the right abutment. USACE evaluated a couple of tunnel alignments, and placing the tunnel control at the upstream, mid-tunnel, and downstream. In all cases, the alternatives proved to be very costly.

In an effort to minimize costs, USACE developed other outlet works arrangements: Modified Spillway With Gate in Slot, Modified Spillway With Spillway Sluice, and Short Spillway Tunnel. The first of these alternatives involved cutting a 37-foot deep by 24-foot high slot in the existing spillway, and providing a gate in the slot. This alternative was dropped by USACE over concerns about gate vibration during operation, potential debris problems, and maintenance and repair of the submerged gate.

3.3 Steel Crest Gate Alternative

As mentioned previously, USACE considered installing a steel bascule gate on the existing spillway crest. As a general policy, USACE does not recommend the use of spillway crest gates for dams that control flows from small drainage basins, which have short times of concentration during flood. The concern is that dam operation personnel would not be able to respond quickly enough during flashy flood events. This is much more of a concern in portions of the country where thunderstorms and flash floods are common. In practice, there are a number of dams with spillway gates in the Northwest that are in small basins and that are operated successfully by USACE or others.

This was reviewed again briefly in the previous study to determine the approximate current cost of a steel gate structure. From the USACE 1982 report, the cost for just the steel bascule gate and operating equipment was estimated to be $2,330,000 (Oct. 1982 prices). Using the ENR construction cost index, the current cost of a steel bascule gate would be approximately $3,975950. In addition to the high costs, there were concerns about debris preventing full closure or opening of the gate, and potential interference problems with the spillway sluice gate outlet works alternative. Other forms of steel gates, such as radial gates, were not examined due to the need for placement of intermediate piers, which would require a significant spillway expansion to accommodate.

3.4 Short Tunnel Concept

In an attempt to minimize costs, the concept of a short tunnel located between the spillway and dam embankment was briefly evaluated by USACE. The concept included an intake tower located in the spillway approach channel with an approximately 165-foot long tunnel exiting into the right wall of the existing spillway chute. This concept appeared to be the least costly; however, it was still not deemed cost effective, and it presented numerous technical concerns. The construction cost of this alternative was estimated to be approximately $3,779,000 in 1989. The total project cost, including engineering and construction management, was estimated to be $9,959,000. Escalated to current 2000 price levels, using the ENR construction cost index, results in a total project cost of approximately $14,042,190. This concept was updated and reevaluated as Alternative 2B2.

4. Current Alternatives

Four basic alternatives are being studied currently, and they are listed below.

Alternative 2B1 – Spillway Sluices

Alternative 2B2 – Short Tunnel with Slide Gates

Alternative 2B3 – Short Tunnel with Submerged Tainter Gate

Alternative 2B4 – Tainter Gates in Spillway Chute

These alternatives were chosen by the USACE and were based on the previous studies by the USACE and P.I. Engineering. The following sections describe each of the alternatives in greater detail. Initially, each alternative was developed for a flood control pool having a minimum elevation of 455 feet and a maximum elevation of 492 feet. This flood storage pool elevation would provide approximately 20,000 acre-feet of flood control storage.

After the initial analysis, it was decided to also look at a couple options for a flood storage pool at elevation 477 feet. This is the elevation of the existing spillway crest. This flood storage pool elevation would provide approximately 11,000 acre-feet of flood control storage. This option was looked at for Alternatives 2B1 and 2B2. It was not looked at for Alternative 2B3, since that alternative is so similar to 2B2. It was also not considered for Alternative 2B4 since the outlet structure for this alternative is sized to convey the entire PMF outflow from the dam and a lower flood control pool level would not affect the design of this alternative. It has also been assumed that low flow releases would continue to be made through the existing outlet works consisting of the multi-level intake and two 24-inch Howell-Bunger valves.

A 3,000 csf capacity at minimum reservoir pool was used for the preliminary sizing of each of the outlet works. The Bernoulli energy equation, as well as the standard equations for flow over a weir and through a gate, was used in the sizing of the project features. A detailed numerical analysis has not been performed for this phase of the studies.

Additional studies that would need to be performed in the next phase of studies would include the following:

- Detailed numerical analysis of the spillway, chute, and outlet works.

- Structural design of outlet works and spillway and chute modifications.
- Development of flood control regulation rule curves.
- Evaluation of any downstream environmental effects related to reservoir operation and flood control regulation.
- Evaluation of reservoir sedimentation and bank stability.
- Assessment of potential downstream scour and bank erosion.
- Determination of freeboard requirements.
- Assessment of downstream fish passage.
- Evaluation of cavitation potential.

4.1 Alternative 2B1 – Spillway Sluices

This alternative would involve excavating out a portion of the existing ogee spillway and installing sluice gates. The spillway approach channel would be excavated down to accommodate the new sluice gates, and the spillway chute entrance would be lowered and widened to allow the passage of the new PMF flow. For the high flood storage pool option, an inflatable rubber weir would be installed on top of the ogee spillway.

A section of the existing ogee spillway would be removed and a new spillway section containing three gated sluices would be constructed. The three sluice gates would each be approximately 10 feet wide and 10 feet high. Bulkhead slots would be provided upstream of the gates to allow for dewatering of the gates for maintenance and repair. Plates 5, 6, and 7 show a conceptual plan and sections for the 20,000 acre-feet option.

The existing spillway approach channel is excavated in rock to an invert elevation of 464 feet. A trapezoidal-shaped channel, approximately 250 feet long, would be excavated within the existing spillway approach channel. The hydraulic efficiency of the spillway structure would be affected slightly by the lowering of the spillway approach channel, but it was assumed in the design of this alternative that the effects would be negligible. The new sluiceway approach channel would have a bottom width of about 40 feet, an invert elevation of approximately 442 feet, and 1 horizontal (H) on 4 vertical

(V) sloping sides. Approximately 10,500 cubic yards of rock would need to be excavated to construct the channel.

The existing spillway chute is located in a rock excavation on the left abutment. The chute bottom converges from a width of 40 feet to 25 feet and has 1H on 4V side slopes. The walls are concrete lined 7 to 13 feet vertically above the invert, with excavated rock side slopes above the concrete lining. In order to pass the full PMF flow, the chute entrance would have to be lowered approximately two feet and widened approximately five feet.

The discharge capacity of the existing uncontrolled spillway is approximately 28,000 cfs at the maximum design pool elevation. However, in a PMF discharge event of 32,500 cfs, the existing spillway crest would be submerged by water backing up from the spillway chute entrance. By lowering and widening the spillway chute entrance, hydraulic control would remain at the gates

The three spillway sluice gates would have a total capacity to pass approximately 9,062 cfs at a reservoir pool elevation of 492 feet. For flood flows greater than the 9,062 cfs, the rubber weir would be very gradually deflated, and flows allowed to pass over the spillway crest. The deflation sequence would be carefully designed to ensure that downstream ramping rates are not exceeded. In the completely deflated position, the full PMF flow would be able to pass over the spillway crest.

A 15-foot high by 130-foot wide inflatable rubber weir would be added to the existing spillway crest for the 20,000 acre-feet option. Inflatable rubber weirs have been used very successfully in North America, Europe, and Asia. The weir consists of a heavy-duty, reinforced rubber body that is anchored to a concrete foundation and inflated with air. The height of the weir can be varied by adjustments of the pressure within the tube. If necessary, the weir can be deflated to allow for unrestricted flow of water over the spillway. Controlled deflation of the weir is by a manual system that is backed up by one or two automatic systems. The automatic systems are by a simple float or bucket system that does not require electricity to operate. The rubber dam inflation and deflation mechanism is very simple in design, with a minimum of moving parts. This provides high reliability, minimizing the possibility of any mechanical malfunctions. The flexible structure of the rubber dam also virtually eliminates the influence of any downstream debris or sediment, allowing the dam to be deflated.

During the PMF discharge, the water surface in the chute would overtop the current concrete lined portion of the walls, but would still be contained within the excavated rock channel. This rock material has been identified as being highly fractured and susceptible to freeze-thaw damage. In order to protect the rock portion of the chute, the rock slopes would probably be lined with shotcrete up to the new PMF water surface profile. The invert of the plunge pool below the spillway ogee crest would also be excavated out and lowered to make room for the new spillway sluices.

For the 11,000 acre-feet option with the flood control storage pool at elevation 477 feet, the rubber weir on top of the spillway crest would be omitted. For this option, the three spillway sluice gates would have a total capacity to pass approximately 7,100 cfs at a reservoir pool elevation of 477 feet. For flood flows greater than the 7,100 cfs, flows would start to pass over the uncontrolled spillway crest. Plates 8,9 and 10 show the plan and section for the 11,000 acre-feet option.

4.2 Alternative 2B2 – Short Tunnel with Slide Gates

This alternative would consist of constructing a short outlet works tunnel in the left abutment of the dam between the existing spillway and dam crest. An outlets works tower with slide gates would be built at the entrance to the new tunnel. The tunnel would discharge into the existing spillway chute, which would be modified to handle the full PMF flow. For the high flood storage pool option, three steel tainter gates would be add to the top of the existing ogee spillway. Plates 11, 12, 13 and 14 show a conceptual plan and sections for the low flood storage pool option, and Plates 15, 16, 17 and 18 show a conceptual plan and sections for the high flood storage pool option.

USACE originally developed this alternative in an attempt to avoid some of the high cost items associated with the spillway sluice design. This alternative would consist of constructing an intake structure just upstream of the right abutment of the existing spillway bridge. The intake would lead to a short tunnel constructed in the rock forming the left abutment of the embankment dam. The intake would have two 8-foot by 11-foot slide gates. Flow would discharge through the tunnel into the existing spillway chute. The outlet tunnel and spillway chute confluence will be a very complex feature to hydraulically design and analyze and a physical model investigation may be required in the final design phase.

Due to concerns that the left abutment rock may be highly weathered or fractured, and thus not very suitable for tunneling, it was assumed that the tunnel would be constructed as a cut and cover structure. A trench would be cut down in stages, with rock anchors being placed prior to the next excavation cut. A cast-in-place concrete tunnel would then be constructed at the bottom of the trench. Approximately 12,600 cubic yards of rock would have to be excavated for the tunnel construction. Concrete walls would be constructed at both the upstream and downstream ends of the trench, and the space between backfilled. New grout curtain holes would be drilled to prevent the flow of water through the dam embankment.

The intake structure would be a freestanding tower with an invert elevation of 438 feet, and a top deck at elevation 497 feet. For purposes of the cost estimate, it was assumed that the tower would be cast-in-place concrete, and a precast concrete bridge would provide access. The tower would be approximately 28 by 30 feet in plan, and would contain the two control gates, two guard gates, and all the necessary hydraulic control equipment. An inclined trashrack would be provided at the tunnel entrance, as would bulkhead slots.

The existing uncontrolled overflow spillway would be modified for the 20,000 acre-feet option. A few different options were considered for providing spillway crest control including an inflatable rubber weir and steel bascule gate. For purposes of costs and preliminary engineering, it was decided to go with steel tainter gates. There would be three steel tainter gates approximately 39.3 feet wide and 15 feet tall on the spillway crest with two concrete piers between. An access bridge would be constructed over the top of the gates and piers. This gate arrangement would likely be more expensive than an inflatable rubber crest weir, but would be considered a more traditional design. The exact arrangement for the spillway crest control could be examined more closely in a later design stage.

The outlet tunnel would be designed to discharge up to approximately 8,000 cfs during PMF with the remaining 24,500 cfs passing over the overflow spillway. The overflow spillway would have a total capacity of approximately 25,400 cfs.

For the 11,000 acre-feet option, the existing overflow spillway would remain as it is with no control gates. For this case, the overflow spillway would have a total capacity of approximately 28,000 cfs. In order for the spillway to pass the full PMF flow of 32,500 cfs, the spillway chute entrance would have to be modified as was assumed in Alternative 2B1.

Alternative 2B3 – Short Tunnel with Submerged Tainter Gate

This alternative is similar to Alternative 2B2 described above. This alternative would consist of constructing an intake structure just upstream of the right abutment of the existing spillway bridge. The intake would lead to a channel constructed in the rock forming the left abutment of the dam. The intake would have a single 16-foot wide by 15-foot high submerged tainter gate. Flow would discharge through the channel into the existing spillway chute. An inflatable rubber weir would be added to the existing ogee spillway. No low flood storage pool options were investigated for this alternative. Plates 19, 20, 21 and 22 show a conceptual plan and sections.

As in Alternative 2B2, the outlet channel would be cut down in stages, with rock anchors being placed prior to the next excavation cut. A cast-in-place concrete lining would then be constructed. Shotcrete could be used above the estimated water line. Approximately 12,600 cubic yards of rock would have to be excavated for the channel construction. A bridge structure would be incorporated to allow vehicles to pass over the outlet channel. New grout curtain holes would be drilled to prevent the flow of water through the dam embankment.

The intake structure would be a freestanding tower with an invert elevation of 438 feet, and a top deck at elevation 497 feet. For purposes of the cost estimate, it was assumed that the tower would be cast-in-place concrete, and a precast concrete bridge would provide access. The tower would be approximately 20 by 45 feet in plan, and would contain the submerged tainter gate, and all the necessary hydraulic control equipment. An inclined trashrack would be provided at the tunnel entrance, as would bulkhead slots.

The existing uncontrolled overflow spillway would be modified, and a 15-foot high inflatable rubber weir would be constructed on top. The outlet channel would be designed to discharge up to 8000 cfs during PMF with the remaining 24,500 cfs passing over the overflow spillway. In order for the spillway to pass the full PMF flow of 32,500 cfs, the spillway chute entrance would have to be modified as was assumed in Alternative 2B1.

4.3 Alternative 2B4 – Tainter Gates in Spillway Chute

This alternative would involve constructing a new outlet works tower at the top of the existing spillway chute. Two large steel tainter gates would control flow through the outlet works tower. The existing concrete ogee spillway would be removed, and the spillway approach channel would be lowered to accommodate the outlet works. This alternative was not analyzed further, therefore no drawings are included.

A concrete control structure would be constructed at the top of the existing spillway discharge chute [SDC]. Plan dimensions of the structure would be approximately 64 feet wide by 94 feet long. The height of the structure would vary from a top elevation of 497 feet at the spillway crest [Station 0+00] to elevation 403 feet at the downstream toe of the structure [Station 0+80]. A 14-foot wide concrete center pier and 8-foot wide concrete side abutments would house twin welded steel tainter gates and gate hoisting machinery. Each tainter gate would be approximately 54 feet high by 17 feet wide. Slots would be provided immediately upstream of the gates for inserting emergency stoplogs. A new concrete overflow spillway would be constructed transverse to the SDC centerline with ogee crest at elevation 443 feet. A new low flow outlet pipe would be installed through the new spillway ogee and the center pier to provide fish passage. Fish would enter the pipe at the upstream sill of the new spillway at elevation 430 and exit into the invert of the new SDC low flow notch at the downstream end of the new control structure at elevation 403.

The existing trapezoidal-shaped SDC is currently incapable of handling the revised PMF of 32,500 cfs. The 285-foot long upstream portion of the existing SDC would be demolished and replaced with the new 95-foot long control structure and a new 190-foot long SDC transitioning from rectangular at Station 0+80 to trapezoidal at Station 2+70. Both reinforced concrete sidewalls of the existing SDC would have to be extended at least 5 feet for the remaining 1,200 lineal feet to the exit into the Skookumchuck River.

The existing concrete overflow spillway has an ogee crest at elevation 477 and is located just south of the low flow intake access bridge and just north of and parallel to the SDC. The majority of this spillway would be demolished and a new curved intake channel would be excavated in the existing bedrock for a distance of approximately 440 feet upstream of the new spillway. The width of the new channel including side slopes would vary from 95 feet wide at the new spillway crest to 70 feet wide at the upstream end of the channel. Since the new excavation would undermine the center pier of the existing access bridge to the low flow intake, the pier bottom would have to be replaced.

Approximately 265 lineal feet on both sides of the SDC rock walls downstream of the new spillway would require excavation and rock bolting. Approximately 70 lineal feet of the SDC south rock wall upstream of the new spillway would also require excavation and rock bolting.

5. Cost Estimates

Cost estimates have been developed for the various Skookumchuck Dam Modification Alternatives. All costs are presented in 2001 dollars and exclude interest during construction. The estimates include contractor's overhead and profit, sales tax, and a construction contingency appropriate to this phase of studies.

Quantity estimates were made from work items and materials for the main components of the proposed design. Approximate unit prices were developed from previous cost estimates by USACE and WSDOT, bid prices from similar projects, quotes from manufacturers and contractors, and from current R.S. Means construction cost guides. Construction work was assumed to be limited to 8 hours a day, five days a week.

For the cost estimates, it was assumed that carefully controlled blasting would be used for all rock excavation. It is not known at this time whether there would be concerns with blasting adjacent to the dam. If mechanical excavation methods are required, excavation costs could increase significantly.

Mobilization and demobilization costs were taken as 5% of the direct cost subtotal. Sales tax was applied only to materials and equipment rental and not to labor costs. Contractor overhead and profit was taken as 25% of the direct cost with mobilization and sales tax added. A 25% construction contingency was then added to come up with a total direct cost.

Direct cost summaries for the selected alternative is presented in Appendix D of the GRR .

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